EXPERIMENTAL RESEARCH ON STRUCTURAL CONCRETE MASONRY WALLS SUBJECTED TO FIRE




Rúben F. R. Lopes[1]   João P. C. Rodrigues[2]   João M. Pereira[3]   Paulo B. Lourenço[4]

ABSTRACT

The construction in masonry is one of the oldest ways of construction. However, the knowledge regarding its structural behavior is still not well consolidated, especially regarding extreme conditions, such as fire. Although the few available studies regarding structural masonry under fire conditions show some promising results, still the available normative documents seem to have some limitations regarding the topic, which highlights the need for further research, especially experimental, in this field.

This paper presents the results of a study on the behavior of structural masonry walls subjected to simulated fire conditions. The specimens were built with three cells concrete blocks, which are common in European and US construction. In the fire resistance tests the walls were subjected to the action of a serviceability in-plane load and the ISO 834 fire curve. Temperatures were measured in the furnace, while temperatures and displacements were measured in the specimens. The obtained results were also compared with the Eurocode 6 part 1.2 predictions, for similar cases.

NOTATION

The following notation is used

$f_{\mathrm{ak}\perp}$ characteristic value of the compressive strength normal to bed joints at normal temperature

$f_{\mathrm{d}\perp}$ maximum design value of the compressive strength normal to bed joints at normal temperature

1 INTRODUCTION

Concrete masonry has been widely used all over the centuries in loadbearing and partition walls. Concrete masonry units are commonly bonded by cement mortars. In Europe, Eurocode 6 states that masonry walls have to meet various requirements when exposed to fire, with the main requirements being **I** for temperature insulation, **E** for integrity to avoid the flow of smoke and hot gases through the wall and **R** for load-bearing capacity. A masonry wall to check a particular fire resistance has to verify one or more than one of the mentioned requirements for a certain minimum period of time.

[1] MSc Student. ISISE, Department of Civil Engineering, University of Coimbra. Portugal. e-mail: ruben.lopes@student.dec.uc.pt.
[2] Professor. ISISE, Department of Civil Engineering, University of Coimbra. Portugal. e-mail: jpaulocr@dec.uc.pt. **Corresponding Author**.
[3] Postdoctoral researcher. ISISE, Department of Civil Engineering, University of Minho. Portugal. e-mail: jpereira@civil.uminho.pt
[4] Professor. ISISE, Department of Civil Engineering, University of Minho. Portugal. e-mail: pbl@civil.uminho.pt

In the last 50 years, there has been several investigations to assess the load-bearing capacity of these elements when exposed to fire. In the 70s, Byrne (1979) conducted 14 fire tests on load-bearing masonry walls with clay brick units varying the height of the specimen and the applied load on the wall. In this investigation it is indicated that the slenderness and the level of applied loading on the specimens have a great influence in the load-bearing capacity of the wall at high temperatures, presenting the more slender specimens smaller resistance.
In the 70s and 80s Lawrence and Gnanakrishnan (1987) conducted a test campaign on 146 full-scale load-bearing walls, with masonry units of different material types and thicknesses. The results of this investigation proved, to some extent, the findings of the investigations of Byrne (1979).
Shields *et al.* (1988) studied the thermal bowing of masonry walls and the influence of the differential heating on the loadbearing capacity of the walls. These authors used tests of reduced scale walls, with calcium silicate bricks. They concluded that there are two fundamental aspects of bowing, restrained and unrestrained, that act independently of each other. Restrained bowing leaded to permanent deformation and unrestrained bowing seemed to be reversible.
In the following years, several numerical models incorporating the knowledge of the investigations carried out by Byrne, Lawrence, Gnanakrishnan and Shields were made, such as the model presented by Nadjai *et. al.* (2003), facilitating the methods of designing of masonry walls exposed to fire and their incorporation into design codes. The problem with these numerical models is that they can only be used in solid masonry units.
In 2006, Al Nahhas *et al.* (2007) presented an experimental and numerical study of fire-exposed load-bearing masonry walls with concrete hollow-blocks. This study identifies the phase-change phenomena of the free water in the constitutive material of the specimen and identifies the existence of plateaus, periods of time without significant change in values, in vertical and lateral displacements and also in temperature on specimen, establishing a relation between the plateaus and phase-change phenomena of the free water evaporation in the specimen.
Nguyen and Meftah (2012) later performed tests on clay hollow-bricks and verified the existence of plateaus in the vertical and lateral displacements and in the temperature of the specimens in this type of walls. Also, they have justified the results with the existence of parasitic bending moments, due to the eccentricity of the applied in-plane loads caused by the thermal bowing of the wall. According to them, these parasitic bending moments are beneficial because they counterbalance the effect of heating, which leads to a prolonged lateral displacement plateau. These moments lead to collapse of the specimen as soon as there is spalling of part of the masonry units.
The research on load-bearing concrete masonry walls subject to fire is still in its infancy. Due to this fact it was decided to carry out a numerical and experimental study on this topic. The study was performed at the University of Coimbra with the collaboration of University of Minho, in Portugal, being the experimental tests conducted at the Laboratory of Testing Materials and Structures of the University of Coimbra.

2 EXPERIMENTAL TESTS

2.1 Experimental Setup

Fig. 1 presents the experimental setup used to conduct the experimental tests, composed of a reaction frame made of HEB 300 steel profiles with a 933 kN hydraulic jack attached to it. This hydraulic jack, used to apply the in-plane loads, was controlled by a Walter + Bai NSPA 700 / DIG 2000 servo-controlled central unit. The fire load was applied by a modular electric furnace. The specimen was bolted to the reaction slab below and, to distribute the in-plane load, a 1.45 m RHS 350x150 steel profile and a 1.98 m HEB 240 steel profile were put on top of specimen, see Fig. 1b). The testing data was recorded using a TML TDS-530 data logger. All lateral displacements and temperature points were measured according to EN 1365-1 (2012) as shown in Fig. 3 using linear variable displacement transducers (LVDT). The vertical displacements were measured on the load distribution beam placed on top of the specimen at 50 mm of each end.

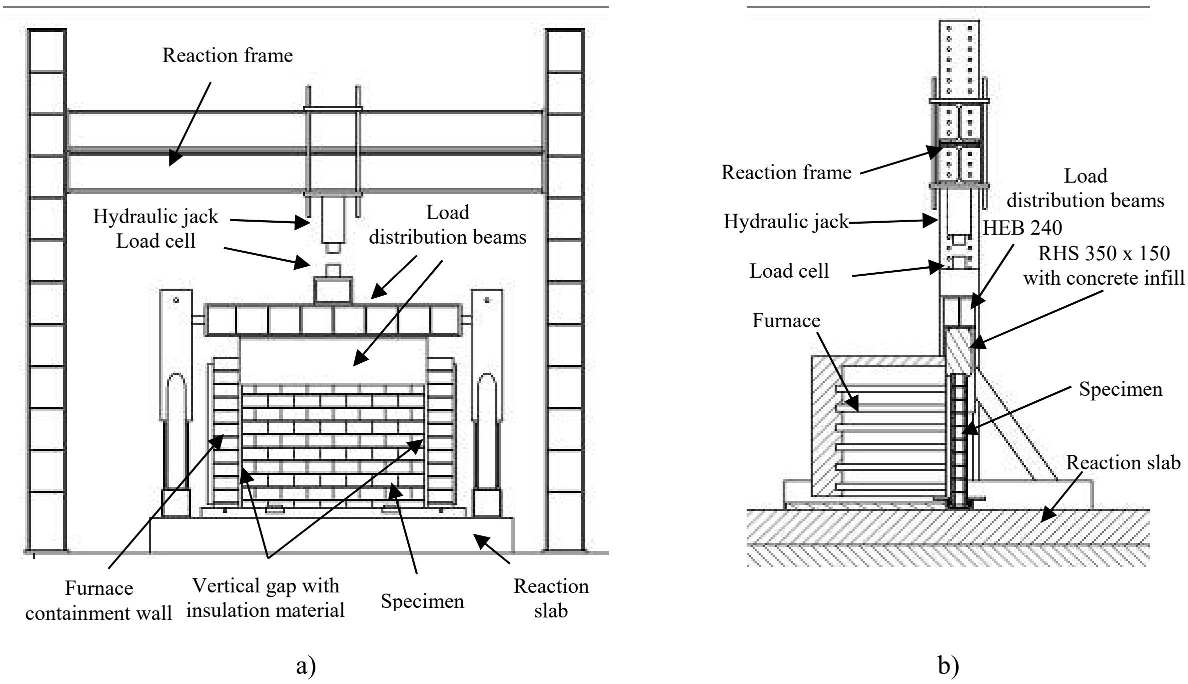


Fig. 1. Experimental setup: a) Front view; b) Longitudinal cut view

2.2 Specimens

The experimental program comprised six load-bearing masonry walls built according to EN 1365-1 (2012) and EN 1363-1 (1999). The walls were made of three-cell masonry units, similar to the ones used by Haach (2009) on his research at normal temperature (Fig. 2).

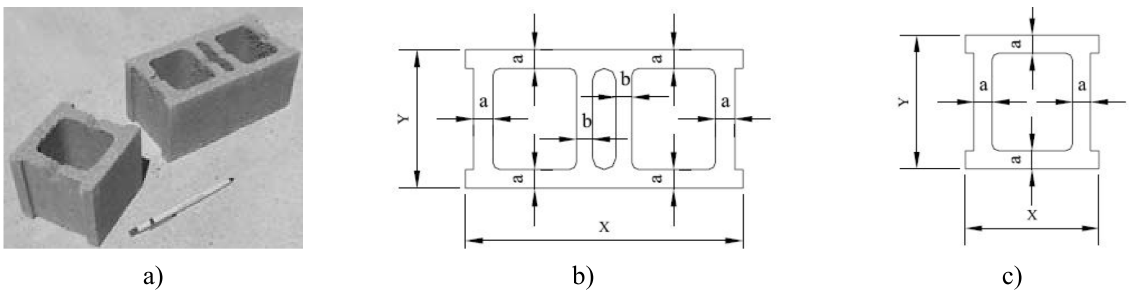


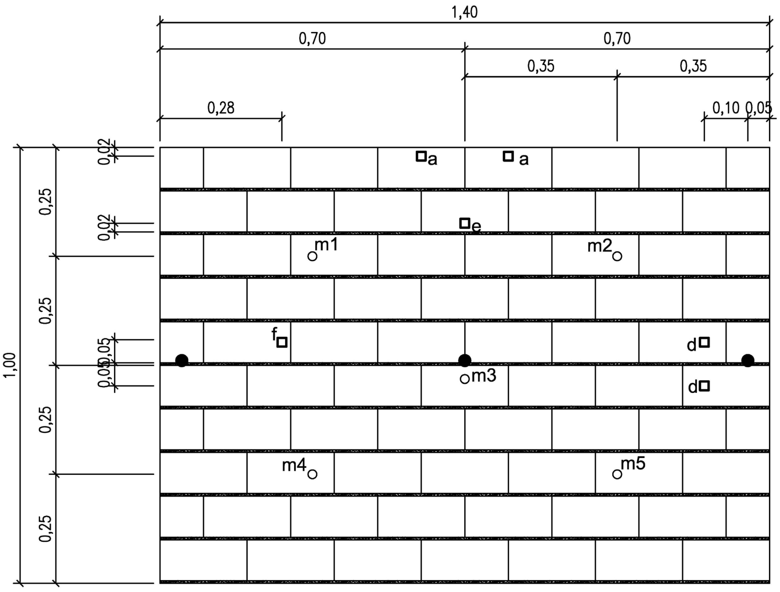
Fig. 2. Masonry Units: a) Reduced scale blocks; b) Block; c) Half block (Haach, 2009)

The masonry units had a scale of 1:2 due to limitations of the load application systems of the Laboratory for applying loads at levels of real scale walls. The dimensions of the concrete units presented in Fig. 2 are given in Table 1. According to the classification proposed in EN 1996-1.1 (2005) these concrete units belong to group 2, due to the percentage, size and orientation of holes.

Table 1. Physical properties of units (Haach, 2009)

	X (mm)	Y (mm)	Z (mm)	a (mm)	b (mm)	Net area of blocks (cm^2)	Area of voids (cm^2)	Percentage of Voids (%)
Block	201	100	93	16	14	110.14	93.92	46
Half-Block	101	100	93	16	-	57.20	46.10	45

In these fire tests, the specimens were composed by seven units in length (1.40 m total) ten courses in the height with 7 mm of horizontal mortar joint (1.00 m total) (Fig 2). The mortar used on the horizontal joints is a commercial M10 mortar, manufactured according to EN 998-2 (2010). The specimens were built in a UPN 160 steel frame for transportation and at the same time create a possibility to fix them in the test rig during the test. To create this connection, the space between the frame and the masonry units was filled with the same mortar used in the horizontal joints.



Legend:

a) at the head of the specimen at mid-width (20 mm below);
d) at mid height of one edge, 150 mm in from the edge;
e) at mid width, where possible, adjacent to a horizontal joint;
f) at mid height, where possible, adjacent to a vertical joint.

□ Maximum temperature thermocouple
○ Average temperature thermocouple
• Lateral displacement point

Fig. 3. Specimen dimensions and positioning of thermocouples and lateral displacement transducers

2.3 Test program and procedures

The experimental campaign comprised three different types of loadings, see Table 2.

Table 2. In-plane loads applied to the specimens

Specimen	Initial Load (kN)	Initial load increase rate (kN/s)	% $f_{ak}\perp$ (EN 1052-1)	% $f_d\perp$ (EC6-1.1)	Load increase rate after 90 minute of fire exposure (kN/s)
1	208	0.5	30	70	0
2	208	0.5	30	70	0
3	319	0.5	46	108	0
4	319	0.5	46	108	0
5	208	0.5	30	70	0.05
6	208	0.5	30	70	0.05

Specimens 1 and 2 were subjected to a load level of 30% of the $f_{ak\perp}$ as proposed by Haach (2009) for this type of masonry walls, applied at a rate of 0.5 kN/s and then exposed to a fire load according to ISO 834 curve until collapse. The in-plane load was kept constant during the fire load exposure.

Specimens 3 and 4 were subject to same experimental procedure as specimens 1 and 2 but with a load of 46% of the $f_{ak\perp}$ as proposed by Haach.

Specimens 5 and 6 were subject at a first level with a load 30% of the $f_{ak\perp}$ as proposed by Haach, then exposed to a fire load according to ISO 834 curve and after 90 minutes of fire exposure, the in-plane load was increased at a constant rate of 0.05 kN/s until collapse. As stated in EN 1365-1 (2012), the vertical edges of all specimens were left with a gap of 25mm to the furnace closure (Fig. 1 a)).

3 RESULTS

3.1 Temperatures

Fig. 4 shows the temperature evolution of specimen 6 throughout its testing, as an example. In the beginning of testing, there is a plateau on the unexposed face temperature points for 15 minutes. Then temperature starts to increase at constant rate, until the 90-100 ° C temperature interval is reached, when the free water in the constitutive material starts to evaporate.

During free water evaporation, there is a temperature plateau at 90-100 ° C for 10 to 30 minutes, depending on position at height. The highest temperature measurement points (a1 and a2) show clearer plateaus, lasting almost 30 minutes, than the lowest points (d1 and d2), which last for 10 minutes. This leads to the conclusion that there may be steam flow through the vertical holes of the blocks and steam accumulates at the top of the specimen, cooling the top of it.

After free water evaporation, a clear thermal gradient between the exposed and the unexposed faces is observed, as furnace temperature and maximum temperature points on unexposed face keep a constant distance between them.

Regarding furnace temperature, the modular electrical furnace has some problems to follow the ISO 834 curve, due to its initial thermal inertia normal on these type furnaces and the high heat capacity of the specimens, despite the special care taken to avoid heat loss on furnace.

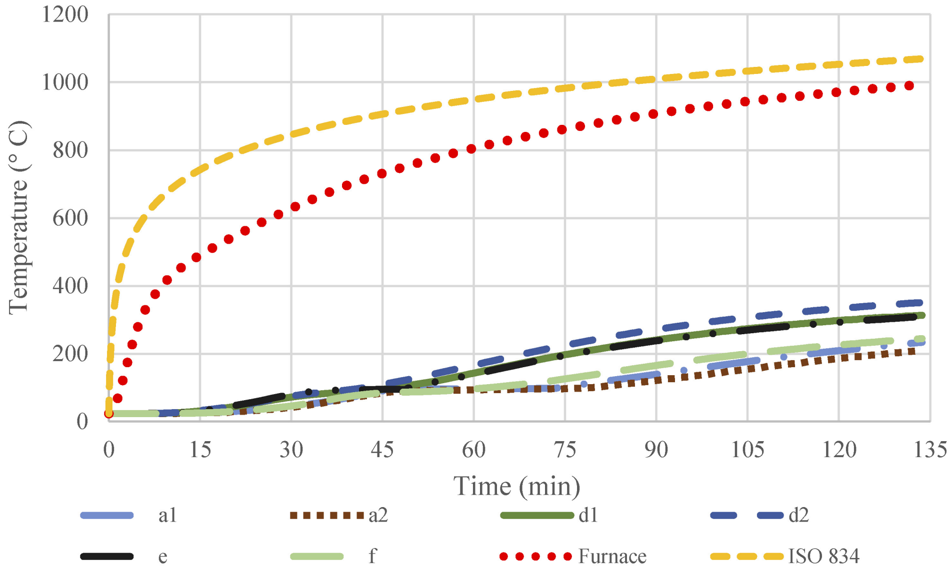


Fig. 4. Temperature evolution throughout specimen 6 testing

3.2 Displacements

Fig. 5 shows vertical displacements of specimen 6 throughout its testing. There are many similarities to other specimens, which will be addressed next. During the first 15 to 30 minutes there was a plateau with small vertical displacements, that had a steady increase rate after this point, up to 60 minutes. From the minute 60 point forward, there was a decrease in the rate of the vertical displacements until 90 minutes. This was also observed in the other specimens, although specimens 3 and 4 collapsed earlier than the minute 90 point (specimen 3 collapsed after 83 minutes of exposure and specimen 4 after 40 minutes) due to higher initial in-plane load. After the minute 90 point, specimens 1 and 2 kept an almost constant value until test stopping and specimens 5 and 6 kept a constant decrease rate until collapse, due to constant rate in-plane increasing load after minute 90 point.

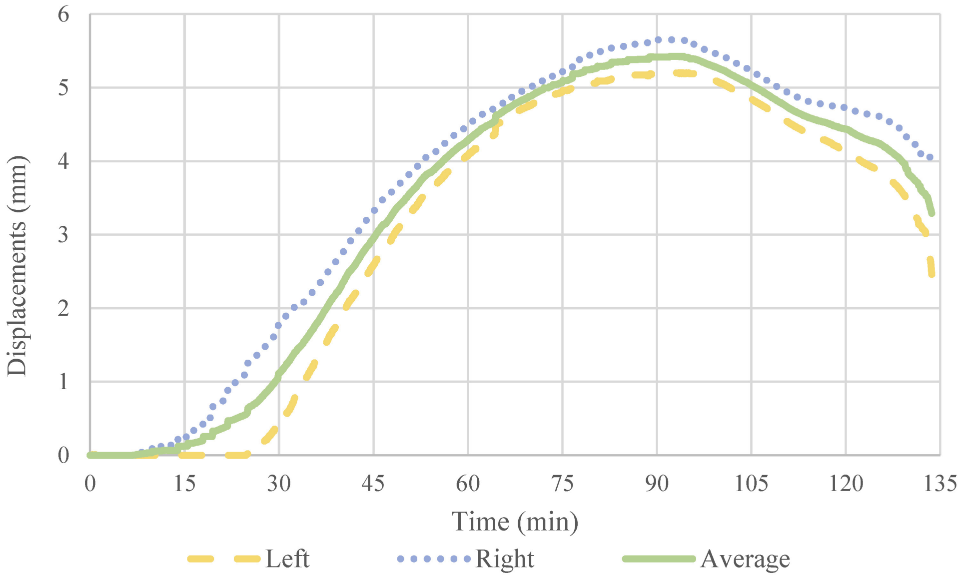


Fig. 5. Vertical displacement evolution throughout specimen 6 testing

Similar behavior occurred in lateral displacements (Fig. 6), with an initial plateau during the first 10 to 20 minutes without significant displacements. In the following 45 minutes, occurred an increase in the lateral displacements, being these displacements towards the furnace. Around minute 60 point, it started a second plateau in the displacements for specimens 1, 2, 5 and 6. Due to higher initial in-plane loading, this second plateau was not observed for specimens 3 and 4, that kept a constant rate increasing lateral displacement until collapse. For specimens 1 and 2, this second plateau was kept until minute 120 point and for specimens 5 and 6 until minute 90 point, due to constant rate in-plane increasing load after minute 90 point. After the end of this second plateau, there was a decrease in the displacements until the end of the test.

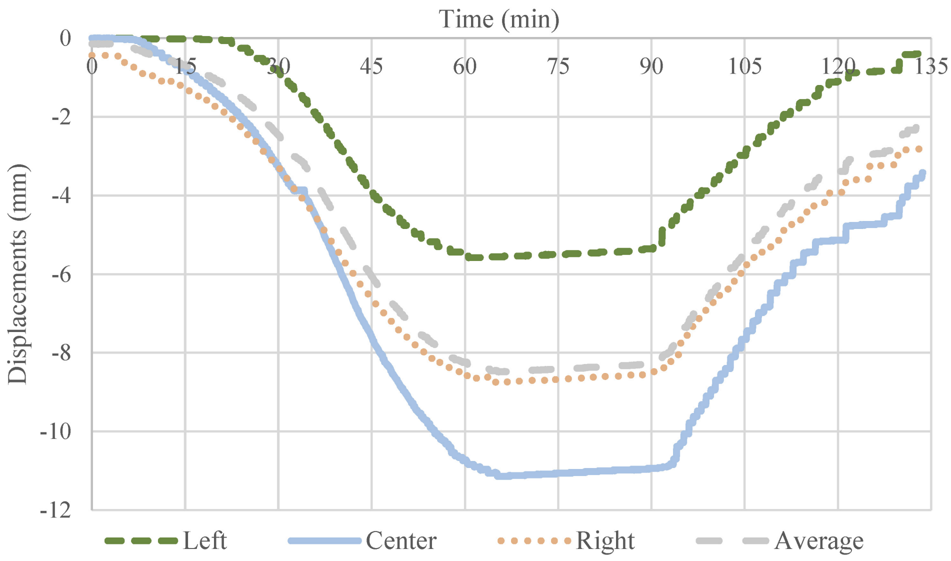


Fig. 6. Lateral displacement evolution throughout specimen 6 testing

3.3 Visual assessment

In all specimens, the first vertical cracks appeared around 30 minutes (Fig. 7 a)), where there was a clear condensation of the steam that is emerging through these cracks, but these cracks were not large enough to allow the flow of hot gases or smoke. These vertical cracks appeared near the center of the blocks, being coincident with the smallest cell in the middle of the block. There were also horizontal cracks that predominate in the area of the mortar joints and diagonal cracks that start at the corners of the specimen and propagate to the center.

There were clear signs of spalling (Fig. 7 b)) on the face exposed to the fire load, but this spalling zone does not go through all the length wall side of the block exposed.

There were also vertical cracks (Fig. 7 c)) at middle thickness of walls, that could be originated by the parasitic bending moments, referred by Nguyen and Meftah (2012) in their experiments, and also by a small recess in the middle of walls' thickness of the block made during the blocks production, which makes a fragile zone in the block. These vertical cracks were clearly seen in specimens 3, 4, 5 and 6 (Fig. 8)

a) b) c)

Fig. 7. Specimen 2 – a) crack pattern of the fire unexposed face; b) spalling on the fire exposed face c) vertical cracks on the width wall of the blocks

The collapse of the specimens 3, 4, 5 and 6 were abrupt (Fig. 8 a)), without major warning signs, like great displacements or sudden loss of in-plane load. The best signs that specimens did to predict the collapse are some small cracking noises coming from the wall.

After cooling, in a visual assessment of debris, is noticeable that the wall center blocks are split in half by the block width, in that fragile zone indicated above.

a) b) c)

Fig. 8. Specimen 4 – a) Sudden collapse of the wall in a test; b) vertical cracking of the blocks in both directions c) block with cracks

3.4 Critical times

Table 5 presents the test results on collapse of the specimens for the different fire resistance criteria, ultimate load level and maximum central out-of-plane displacements.

Table 5. Test Results

Specimen	Time to failure				Ultimate load level (kN)	Maximum central displacement	
	I (min)		**E** (min)	**R** (min)			
	Average temperature	Maximum temperature	Loss of integrity	Structural collapse		Reading (mm)	Time (min)
1	80	72	-	-	-	5.52	74
2	73	67	-	-	-	5.80	79
3	-	-	-	83	319	9.52	80
4	-	-	-	40	319	10.74	40
5	83	82	-	106	273	11.58	68
6	68	65	-	134	421	11.14	66

4 CONCLUSIONS

The following conclusions can be drawn from this investigation

- Masonry is a very heterogeneous material. This shows the reason for obtaining divergent results in terms of time and in-plane loads for identical situations.
- The masonries tested provide good insulation to heat for at least 60 minutes (the lowest time value obtained is 65 minutes). The EC 6 part 1.2 recommends for a masonry wall of blocks with EI 60 criteria a minimum thickness of 70 to 100 mm.
- Considering only the R criteria, the same slenderness in real-scale wall equal to the tested walls and the same load index, it is likely that this block in full-scale (w = 200 mm) is capable of reaching a R 120 criteria. Eurocode 6 requires for a wall of the same material, a thickness of 300 mm. This conclude the Eurocode 6 is being very conservative at this point.

ACKNOWLEDGMENT

The authors grateful acknowledge the support of Costa & Almeida, Lda. and SECIL Argamassas S.A. in the provision of the concrete blocks and mortar for the specimens

REFERENCES

[1] Byrne S. (1979). *Fire resistance of load-bearing masonry walls.* Fire Technology 15-3 (1979), doi:10.1007/BF01983196, pp 180-188.

[2] Lawrence S., Gnanakrishnan N. (1987). *The Fire Resistance of Masonry Walls - An Overview.* First National Structural Engineering Conference 1987, Melbourne, Australia, 26-28 August 1987, pp 431-437

[3] Shields T. J., Connor D. J. O., Silcock G. W. H., Donegan H. A. (1988). *Thermal Bowing of a Model Brickwork Panel*. International Brick/Block Masonry Conference, 8th, Dublin, Ireland, 1988, pp 846-856

[4] Nadjai A., O'Garra M., Ali F. A., Laverty D. (2003) *A numerical model for the behaviour of masonry under elevated temperatures.* Fire and Materials 27-4 (2003), doi: 10.1002/fam.824, pp 163-182

[5] Al Nahhas, F, Ami Saada R, Bonnet G, Delmotte P. (2007). *Resistance to fire of walls constituted by hollow blocks: Experiments and thermal modelling.* Applied Thermal Engineering 27-1 (2007), doi: 10.1016/j.applthermaleng.2006.04.017, pp 258-267

[6] Nguyen T. D., Meftah F. (2012). *Behavior of clay hollow-brick masonry walls during fire. Part 1: Experimental analysis.* Fire Safety Journal 52 (2012), doi:10.1016/j.firesaf.2012.06.001, pp 55-64

[7] EN 1365-1, *Fire resistance tests for loadbearing elements - Part 1: Walls,* European Committee for Standardisation, Brussels, Belgium, 2012.

[8] EN 1363-1, *Fire resistance tests - Part 1: General requirements,* European Committee for Standardisation, Brussels, Belgium, 1999.

[9] EN 1996-1.1, *Eurocode 6 – Design of masonry structures, Part 1.1: General rules for reinforced and unreinforced masonry structures,* European Committee for Standardisation, Brussels, Belgium, 2005.

[10] EN 1996-1.2, *Eurocode 6 – Design of masonry structures, Part 1.2: General rules Structural Fire Design,* European Committee for Standardisation, Brussels, Belgium, 2005

[11] EN 998-2, *Specification for mortar for masonry. Masonry mortar,* European Committee for Standardisation, Brussels, Belgium, 2010

[12] EN 1052-1, *Methods of test for masonry: Part 1 – Determination of compressive strength*, European Committee for Standardisation, Brussels, Belgium, 1999.

[13] Haach V. G. (2009). *Development of a design method for reinforced masonry subjected to in-plane loading based on experimental and numerical analysis*. PhD Thesis. University of Minho, Portugal.